Geosynthetics and Reinforced Soil Structures
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Lecture - 16
Design Example of Reinforced Soil Retaining Walls – II

Good morning students. The previous lectures we have looked at the design of a retaining wall of six meter site with a horizontal backfill, and let us continue the same example by considering additional forces that raise from scismic activities.

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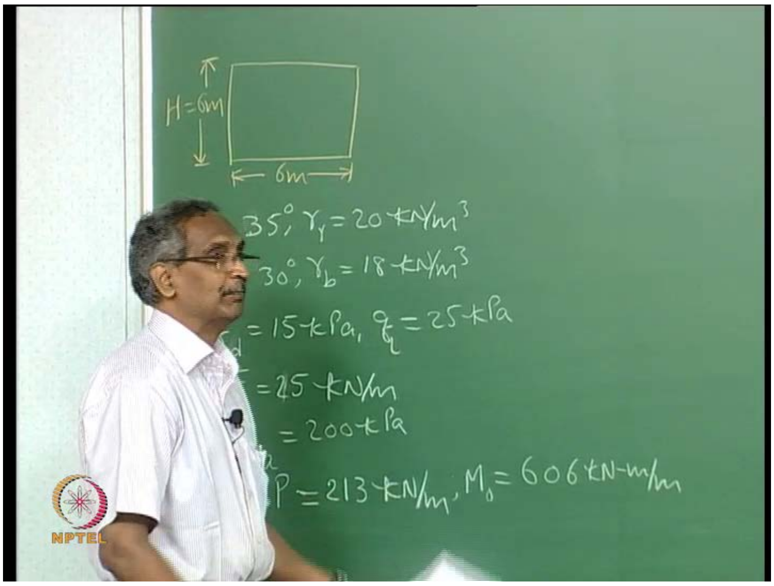

Just to recap the height of previous wall was 6 meters, and the length of the reinforcement that we have calculated was 6 meters, that length of 6 meters was adequate to satisfy the factor of safe in its lateral sliding over turning, and then the bearing pressure and so on. And so the properties that we had assumed, let me just write. We assumed the reinforced fill as having 35 degrees friction angle, and unit weight of 20 kilo Newtons per cubic meter, and the backfill soil has a friction angle of 30 degrees, and unit weight of 18 kilo Newtons per cubic meter. And our the cube dead load was 15 k P a, then the q live load was 25 k P a, and then there was a horizontal load of 15 kilo newtons sorry, 25 kilo Newtons per meter, and our q n a was 200 k P a, and the calculated forces were p of 213 kilo Newtons per meter, and M naught is 606 kilo Newtons.

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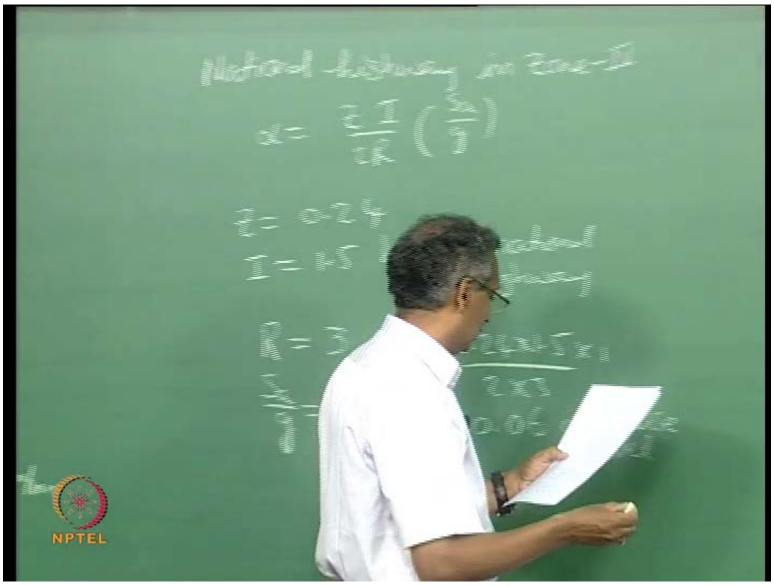

And now let us consider some size forces and let us assume that we are constructing a national highway. Let say that our retaining wall is going to be part of national highway in zone four and our the alpha that is the base acceleration factor is z I by 2 R Sa by g. And the z is the zone factor, we can get the value as 0.24 from the table that we have, and then the importance factor I is 1.5 being a national highway, and then the response reduction factor R we can take it as highest five, because where soil structures are highly ductile, but just to be on the safe side to have a higher conservativeness I am assuming [res/response] response reduction factor of 3.

And our S a by g that is the spectral acceleration coefficient lets a function of the time period of the structure and assuming this once again, bit on the higher side that is on the conservative side as 1.0 corresponding to the amplification with soft soil in the foundation. So, once you substitute all these parameters our alpha is, so at the base level our alpha is 0.06 that is see somewhere here our alpha is 0.06, and then how does it translate into the reinforced fill and that we call as amplification.

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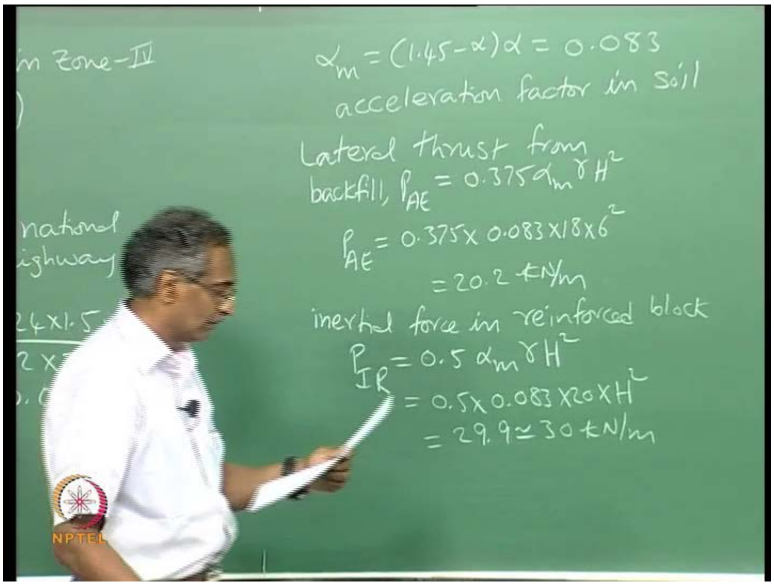

And that amplification alpha m is given as 0.083 this we call as the acceleration factor in the soil and once you have this alpha m that is the acceleration factor in the soil. We can calculate the different seismic forces that is lateral thrust from the backfill soil. And then the inertial force that is developed within the reinforced fill, so the lateral thrust from backfill see notice, that the unit weight that I have used is 18 that is corresponding to the backfill soil and this 0.375 tends the alpha m of 0.083 and the height of the wall is 6 meters and similarly, the inertial force see in reinforced block. So, the inertial force comes out as nearly 30 kilo newtons per meter length of the wall in the perpendicular direction.

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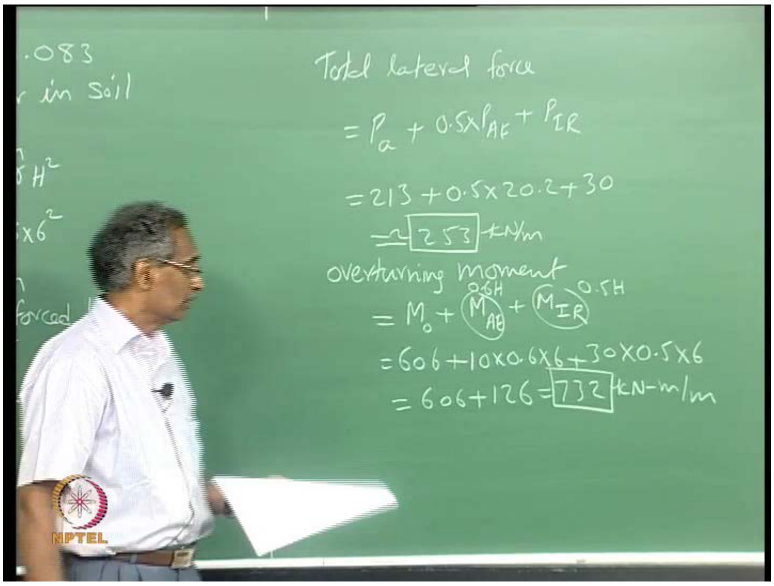

And our the total force total lateral force is this sum total of this static force plus 0.5 times Pa e plus P IR. As we discussed earlier we assume that the lateral thrust and the inertial force. They do not peak at the same time and because of that we consider only fifty percent of the lateral thrust. So, this is equal to 213 plus 0.5 times 20.2 plus 30, this comes to 253 kilo newton per meter, and then the overturning moment. The total overturning moment is this static over turning moment plus the over turning moment, because of this backfill thrust, and then the over turning moment, because that the inertial forces. So, this is 606 plus approximately 10 times 0.6 times six plus 30 times 0.5 times 0.6.

So, here our inertial force is assume to act it 0.5 H whereas, the these at the backfill thrust is assumed to act it 0.6 H. So, that is what i have done here, and the we assume that only fifty percent of the backfill thrust is considered as part of our calculations. So, this whole thing comes to 606 plus 126 equal to 732 kilo newton meter per meter and these are the forces that we have let me just see, where is our…

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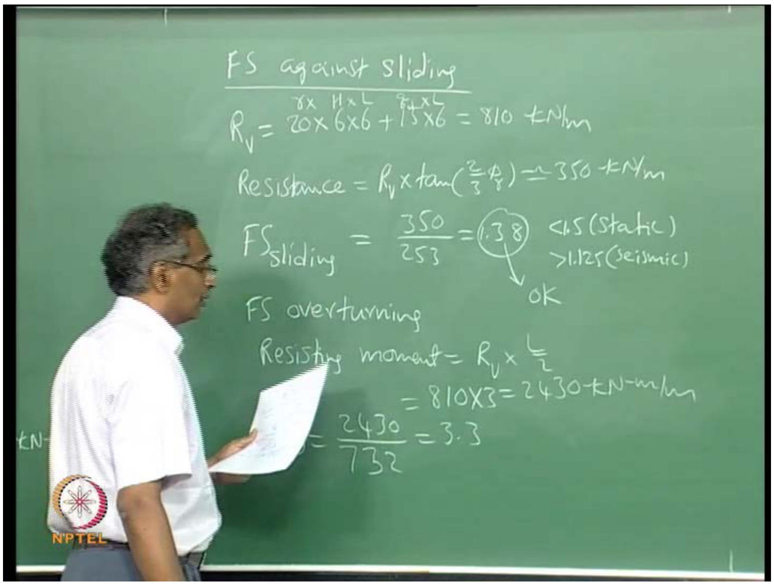

So, this the lateral force by considering the earthquake excitation is 253 kilo newtons per meter, then the overturning moment is increased to 732 and now, let us estimate the factors of safety. So, the fact of safety against sliding as we have [consider] as we have discussed earlier. We neglect the affect of live load for the resistance force means, our Rv is just simply the 20 that is the unit weight of the reinforced fill times the height of the wall and times the length.

So, this is the H times L plus the 15 times 6 that is the q d times L that is 810 kilo newton per meter, then the resistance is approximately 350 kilo newtons per meter and. So, our fact or safety against sliding is 350 divided by 253, this comes out as 1.38, as we have seen in earlier lecture that our block length is very high 6 meters, because of the pores upgrade conditions our q n a is only 200 K p a

and to maintain the bearing pressure within the limits. We have to increase the length of the reinforced block, in spite of that our fact of safety. After considering the seismic comes to 1.38 which is less 1.5, but that is the static greater than 1.125 seismic. So, it is. So, this is we can say that this length of the reinforcement is satisfactory, as far as the seismic ability is concerned then overturning. So, our resisting moment is just simply r v times L by 2 that is the resisting moment is 2430 kilo newton meter per meter. So, our f s is 2430 divided by 732 that comes to 3.3, which is much more than, what we requires. So, it is safe.

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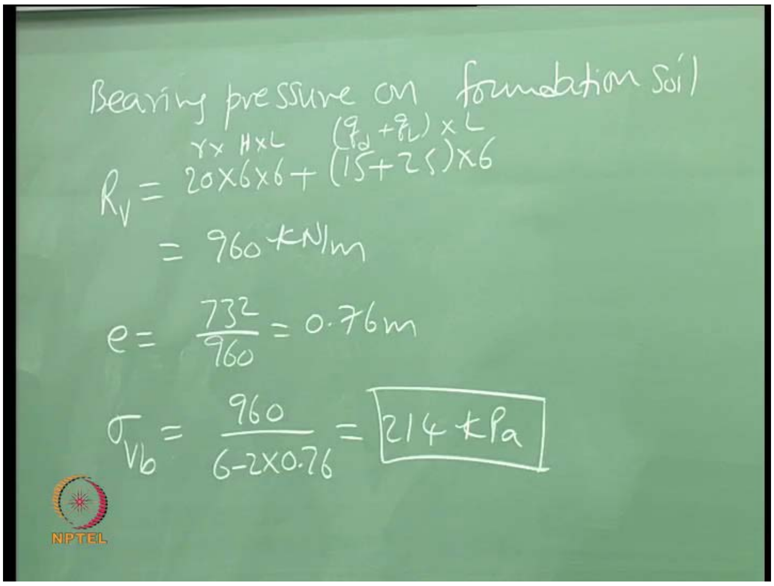

So, let us also check for the bearing pressure or foundation soil, and for calculating the bearing pressure. We include all the loads that is the self rate, the dead load, and the live load. So, that is 20 times 6 times 6 plus 15 plus 25 times 6 that is the height times L. So, our overturning moment including the seismic effects is 732 this divided by the total weight is 960 that gives us the extremity e of 0.76 meters. So, our sigma V b is 960 divided by 6 minus 2 times it is approximately equal to 214 kilo pascals, and this pressure should be less than the seismic bearing capacity of the soil, and the seismic bearing capacity can be estimated in several ways. That is beyond the scope of this course. I will not discuss, but you can refer to some text books the deal with the seismic bearing capacity of the soils.

So, just to summarize this particular retaining wall of 6 meters, having a length of 6 meters reinforced block is stable not only under static conditions, but also under the assumed seismic conditions. For zone four it is safe enough, because it has adequate fact of safety against lateral sliding, and then a adequate fact of safety against overturning, and then the bearing pressure is a 214, which is slightly more than the static pressure of 202 the actually here.

I just assume that the friction angle, the mobilist friction angle even during the seismic event is the same pi r, once again that could be determine for seismic condition, because there are many soils that behave differently under static loading, and under seismic type loading and that has to be once again assist for the particular soil, that we have at the site.

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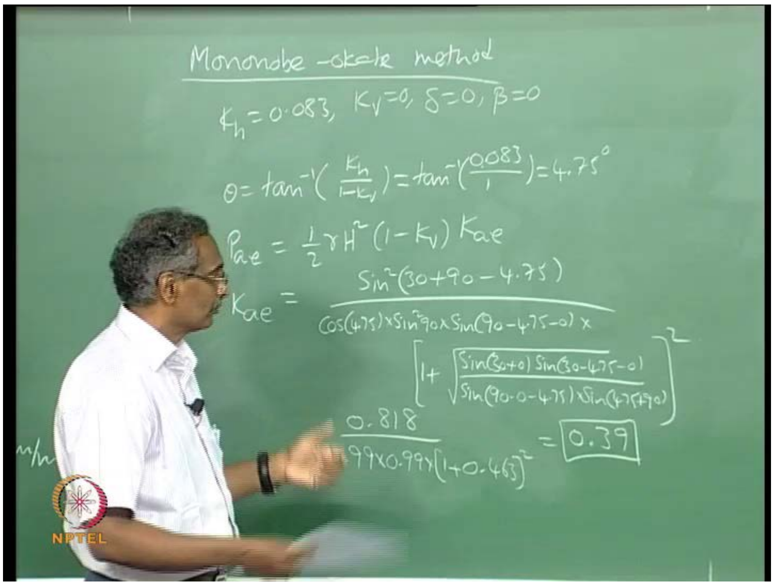

Now, let us estimate the same seismic force using the mononobe-okabe method and this requires a seismic coefficient K h and the K v and as I start. Let us assume that the K h is equal to the same factor that we have that is 0.083, and let us say that our K v is 0, and delta is 0, and beta is also 0, and our theta is tan inverse k h by 1 minus K v, that is the theta is tan inverse of K h divided by 1 minus K v, that comes to 4.75 degrees and our the force is calculated as one half gamma h square times 1 minus k v times K a e, where our K a e I will just directly substitute the different quantities, that we have and this formula .

So, substituted the different value the friction angle, delta, alpha, and beta, and then of course, the theta. So, we get a value of 0.818 divided by 0.99 times 0.99 times 1 plus 0.463 that comes to 0.39. This is the coefficient of earth pressure by including the seismic factors, and this tatic value was one third that is 0.3. Now, it is increased the slightly to 0.39.

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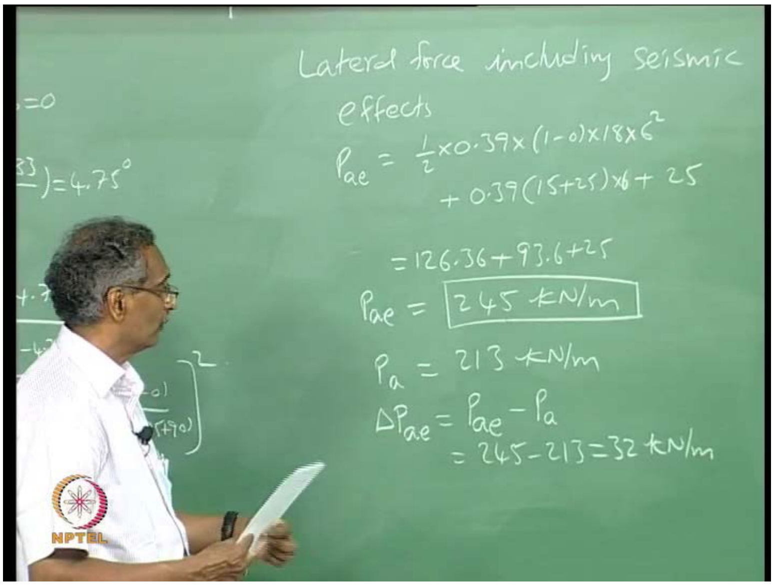

Now, let us estimate the lateral force including the seismic effects, plus see the first term is the gamma h square effect that is the self weight effect, the second term is the effect of the uniform searcher. And the third one is the lateral force that is acting at the top of the wall. So, this whole thing comes to 126.36 plus 93.6 plus 25, that comes to nearly 245 kilo newtons per meter, and we know that the p a was 213. So, our delta p…

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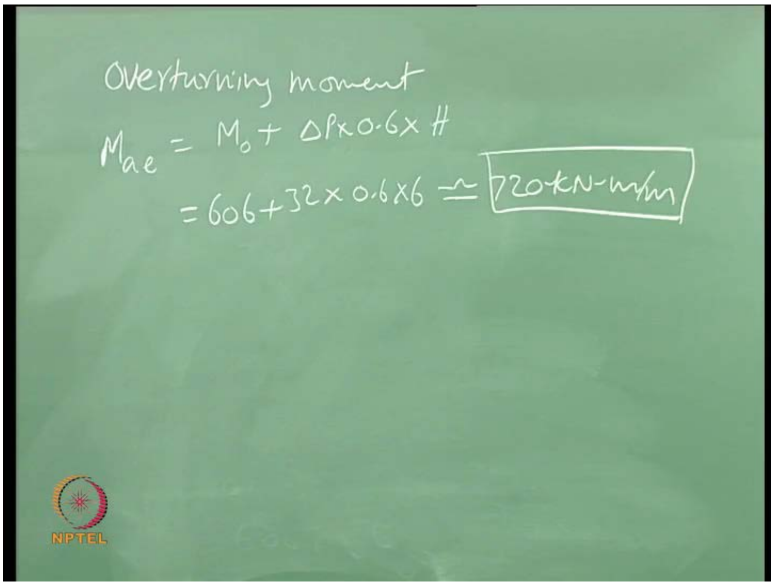

So, the delta p is the sum total of the seismic force minus the sum total of the static force, and that comes to 32 kilo newtons per meter, and our overturning moment. The overturning moment is the static overturning moment plus the delta p times the liver arm that is the liver arm is assumed as 0.6 times the H.

So, this is 606 plus 32 times 0.6 times 6, and this comes to nearly 720. So, our the earthquake overturning moment is 720. So, we can calculate the fact of safety against sliding and overturning, which will come out as very similar to what we had earlier by using the f h w a formula and before we do that? Let us see the effect of vertical acceleration, because in the federal highway administration code we neglected the vertical acceleration.

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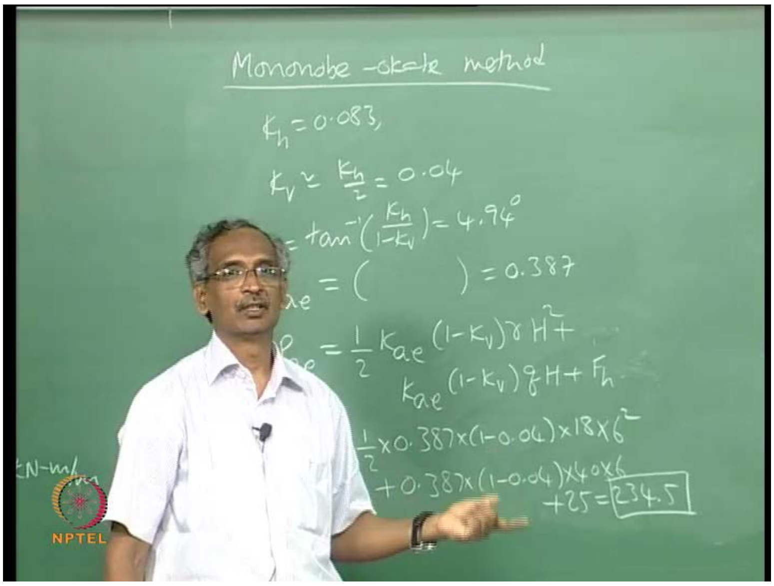

We considered only the horizontal acceleration, and then the first attempt in mononobe-okabe method. We have neglected the vertical acceleration now, let us consider the vertical acceleration and estimate the same forces. So, most of the measured earthquake data is for horizontal seismic factors and the vertical factors are usually not measured, but a good estimate will be the K v of K h by 2. Let us say that we assume the K v to be 0.04 and now our theta is tan inverse K h by 1 minus K v, that comes to 4.94 degrees and our K a e is well. We can substitute all the factors as before and now, we will substitute 4.94 theta instead of 4.75.

And we will see that this comes out as 0.387 slightly smaller than before earlier. We got 0.39, now we have 0.387 and our P a e is one half plus by considering the effect of all the factors like the self weight then this search and horizontal loads. We can calculate the loads like this and that comes to sorry, it should be there simply F h. So, our lateral thrust force now, is only 232 as against 253 that we have calculated earlier by neglecting the effect of vertical acceleration.

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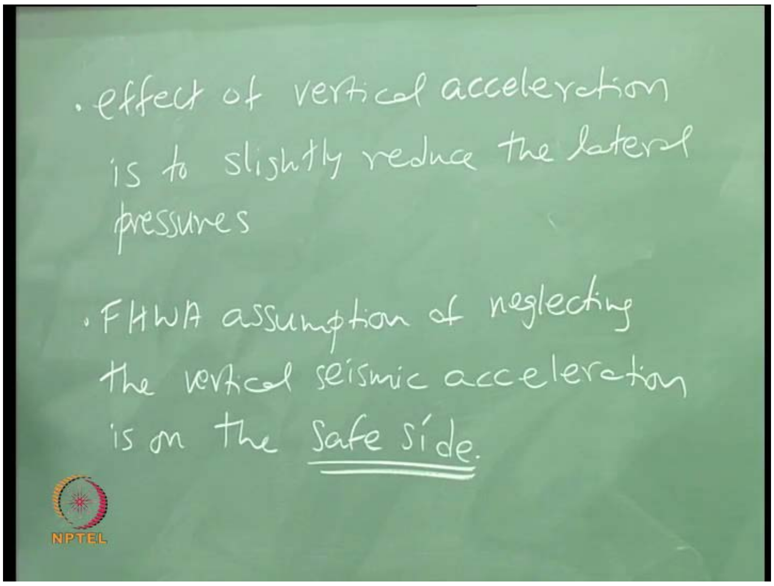

So, we can actually see that the effect of vertical acceleration is to slightly reduce the lateral pressures and consequently, when the overtone moment is also reduced and now, we realize the importance of neglecting the K v value and the F h w a, because anyway the by neglecting, it we are on the safer side that is we design our structure for higher forces the active forces and also the turning moments.

So, we can conclude the f h w a assumption of neglecting the vertical acceleration am underlining this safe side, because we always want to design with a on the safer side rather than on the unsafe side, because there are. So, many unknown factors even the earthquake factors that we consider that is 0.08 in the horizontal direction, and then 0.04 on the vertical direction. There only some empirical values and in the particular scenario that, we have we wrote exactly what is the operating acceleration factors and.

So, on… So, we when we need to assume something we always assume. So, that our design is bit more safer than, what it would have been, if we had assumed some [con/conservative] un-conservative values. So, this is how we calculate the stability under all the external forces. That is the self weight forces the surcharge, and then the forces, because of the schismatic activities and basically, the external stability tells us what should be the length of reinforce block and now, the next step is how to decide the vertical spacing.

And the number of layers and then the factor of safety against the pullout, and then rapture and let us say do those calculations. Now, well we need certain minimum data that is the, what is the type of reinforcement? Whether, it is steel or polymeric, and then what should the strength of reinforcement layers and. So, on, because we do not know exactly unless we do the calculations. We will not know, but then even before we do the calculations. We need to assume some minimal things, and some of the minimal things that we require is the thickness of the compaction layer.

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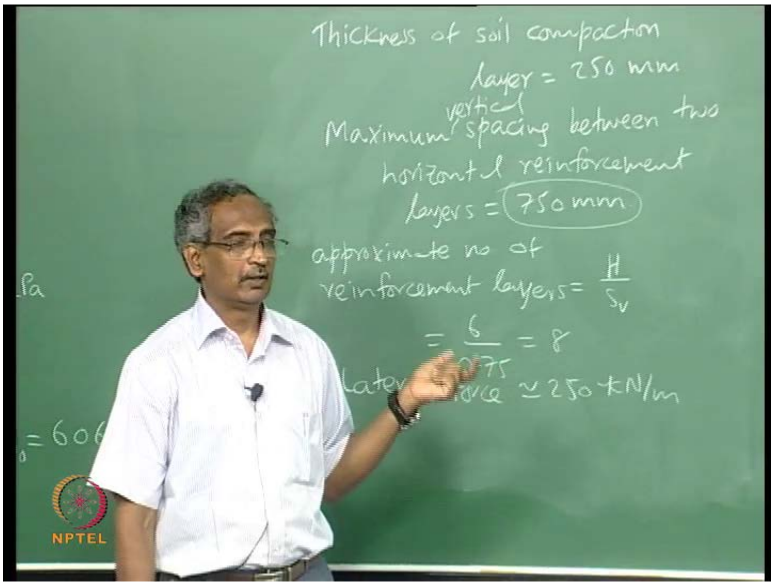

Let us assume that the soil is compacted in layers of 250 millimeters at the side, because this is a very important input for our designs, because our reinforcement layers there always provided at multiples of this compaction layer thickness. So, because our compaction layer thickness is 250 millimeters. This spacing’s could be 250 millimeters 500,750 and so on. And then the other thing is what is the maximum spacing’s, I will be more specific.

What is the maximum vertical spacing between two horizontal reinforcement layers? Most of the design codes they recommend the maximum spacing in the range of 750 to 1 meter and beyond 1 meter. You can design the wall, but then the overall factor safety is ensured, but then locally there could be some failures in between the two reinforce the layers see. If, you have very large spacing the effect of reinforcement may not spread to interior path of the soils.

And, because of that there could be some localized failures and to minimize that the, if i still have a administration they put a limit on the spacing that we can have and let, us assume that for our design the maximum spacing allowed is 750 millimeters. So, the if this is… So, the approximate number of reinforcement layers is height times s v that is 6 by. So, we get about we get 8 layers. So, that is our number of layers, if you follow just the minimal spacing is 8.

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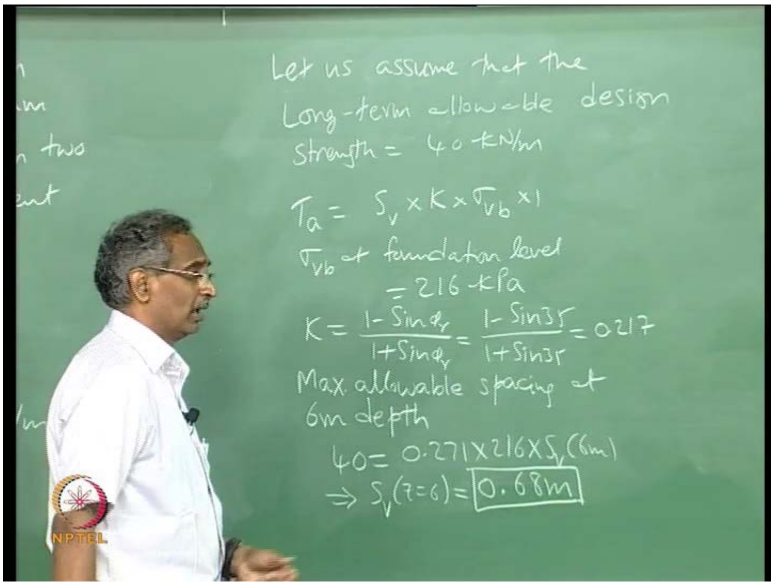

And now our lateral force is approximately a 250 kilo newtons and so since we know that we need to provide the minimum 8 number of layers. We can choose the reinforcement such that, we satisfy the supplying of. So, much of resistance force that is 250 kilo newtons per meter and. Now, let us assume that the t a that is the long term. Let us assume that we provide the reinforcement layer that has a long term allowable design strength of 40 kilo newtons per meter. That is after we account for the crepe reduction factor installation damage environmental degradation factors.

And then other factors the strength that we have is 40 kilo newtons per meter and now, we can choose the vertical spacing and then another requirement that is given in the federal highway administration is that. There must be one layer of reinforcement as close to the top of the wall as possible, because most of the loads they are applied at the surface level and the effect of surface loads is obviously, maximum very near to the surface and. So, there has to be one layer of reinforcement as close to the surface as possible in this particular case that is 250 millimeters.

And. So, we can say that our reinforcement layer the top most reinforcement layer is provided at 250 m, and then what is the spacing? The spacing is obtained by y, this the t a is s v times k times sigma v b times one, and if you assumed that it is continues reinforcement layer like for example, a geogrid and the we can consider unit length in the perpendicular direction. So, we can write our in a quality that is the load transferred into the reinforcement layer is equal to the tensile capacity of the reinforcement layer.

So, our sigma v b at foundation level is 216 k p a and. So, our k is assuming that the reinforcement is a geo-synthetic reinforcement polymeric reinforcement a flexible type. Our lateral thrust pressure constant within the reinforce fill is equal to k a that is 1 minus sine phi by 1 plus sine phi and that r with phi r that is 1 minus that is 0.217. So, the maximum allowable spacing at 6 meters depth is 40 is equal to 0.271 that is the k a and sigma v b is 216 s v of 6. So, this gives is the allowable spacing at a depth of 6 meters comes to 0.68. It is neither 0.75 or not 0.5. So, the allowable spacing's are in multiples of 250 m and the closest allowable spacing lower than 680 millimeters is 0.5.

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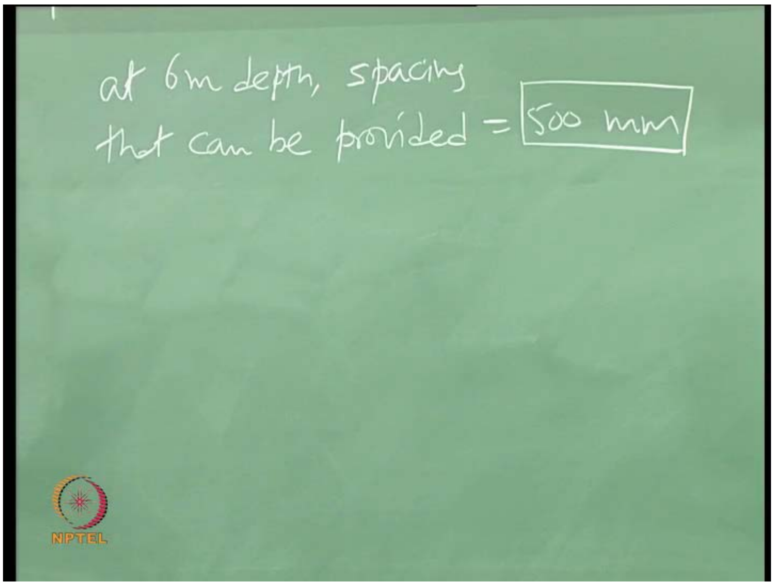

So, we can say that at 6 meter depth the spacing, and let us also calculate what is the spacing? That is allowed some other depth at let say 5 meters depth.

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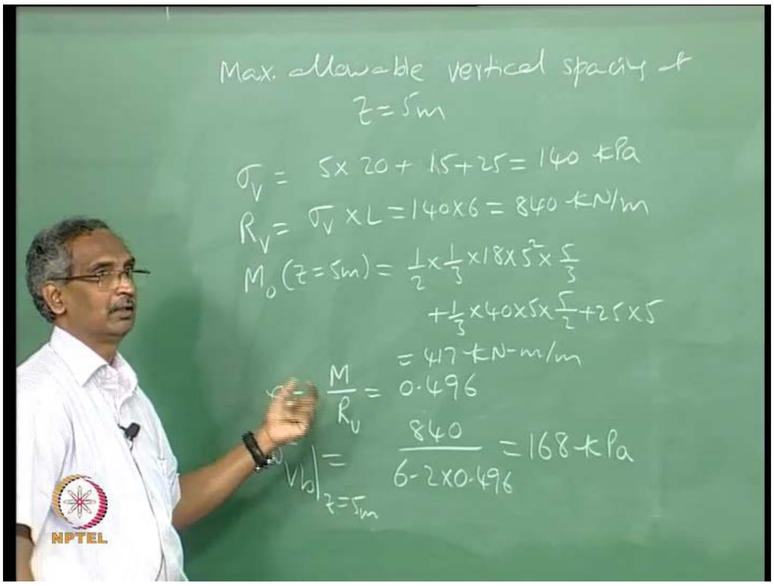

So, let’s repeat the same calculations to calculate the vertical spacing at 5 meters depth. So, our sigma v at 5 meters depth is 5 times 20 plus 15 plus 25 that is 140 k p a. So, our r v sigma v times l that is 140 times 6 that is 840 kilo newtons per meters, and m naught at z is 5 meter is 1/2 of 1/3 18 times 5 square times 5 by 3 plus 1/3 times 40 times 5 times 5 by 2 plus 25 times 5, actually I am not explaining all these things.

Because, we have done similar calculations earlier to estimate the bearing pressure instead of 6 meters. I am now using only 5 meters to estimate the overtone moment at 5 meters depth that is 417 kilo newton per meter. So, our e is m by r v that comes to 0.496. So, our sigma v b at z is 5 is 840 approximately our sigma v b is 168 k p a, as a positive 216 k p a at 6 meters depth.

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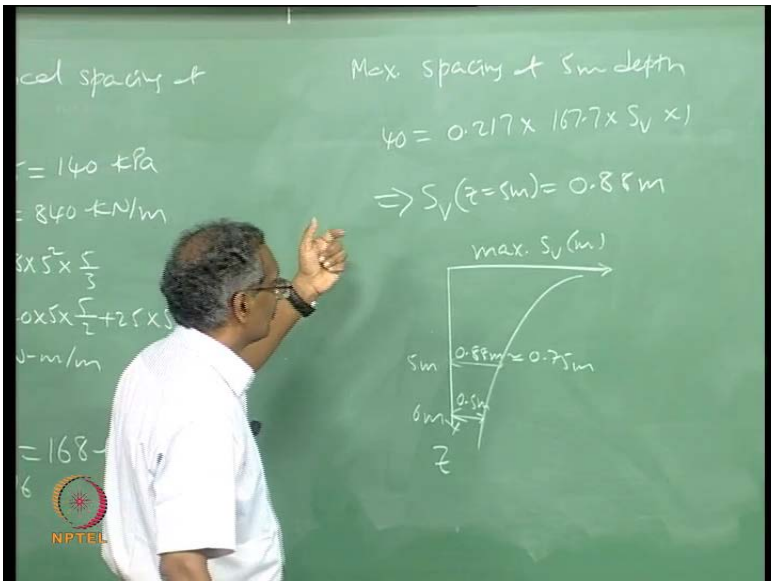

So, the spacing maximum spacing at 5 meters depth is we can get it as 40 is 0.217 times 167.7 times s v times 1 sp our s v at . So, at 5 meters depth the maximum allowable spacing is 0.88 meters. So, that means that we can provide at a spacing of 750 m at 5 meters depth. Now, we can construct our reinforcement layout basically, if you plot a graph between the z on the y axis and maximum s v, we get a graph something like this.

And we have seen that at the at 6 meters depth, we need 5.5 meters and at this is at 6 meters, 5 meters, 0.8 meters that is 0.75 meters and as you go above this spacing may exceed 0.75 meters and to optimize the materials say, when your spacing comes to very large value. Let, us say some two meters what do we do we cannot provide the reinforcement layers at 2 meter spacing, because we may provide or we may provoke localized failure and to optimize, we can use a reinforcement that is of lower strength or just simply use the same reinforcement material.

But at the same [spa/space] or the maximum allowable spacing of 75 m and in this particular case, let us keep the reinforcement the same type over the full depth, because in terms of the construction that becomes more easy. Because if use supply let’s say drawing with stringer reinforcement at the at the base level and the weaker reinforcement at the top level and if the construction supervisor is not that careful and reverses the placement of the reinforcement, that is the weaker layers at the bottom and the stronger layers at the top what will be the result that will be assured disaster.

Because the reinforcement layers at the bottom they will just simply rapture and. So, in order to make the construction supervisor's job more simple. We some companies provide material of the same strength, but then if there is a good constriction supervision. We can do lot of optimizations and then we recommend different strength reinforcements are different depths, but then this being only a class example. I am not going through all those pains, and I am just simply using the same reinforcement layer all over the height of the wall, our height of the wall is 6 meters.

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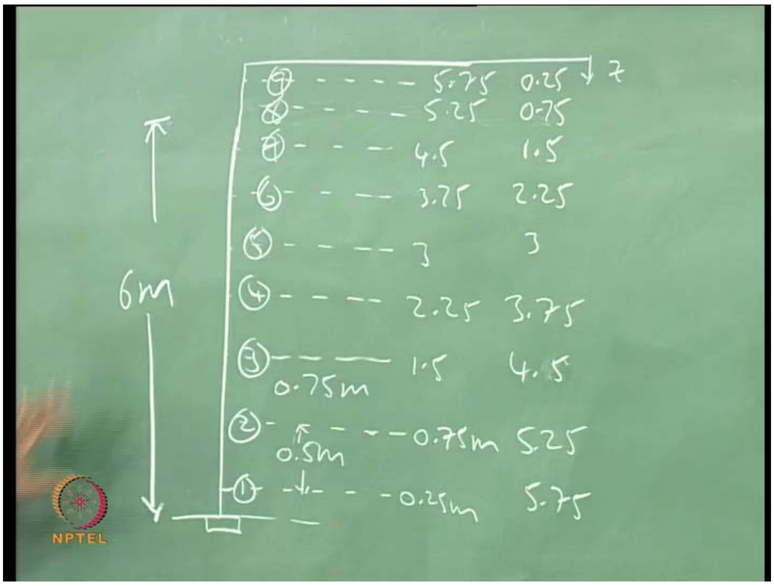

And let us start placing the reinforcement layers from a height of 0.25 meters, above the footing level above the above our leveling pad and our next layer at is at placing of 0.5, 0.75 meters, that i 0.75 meters, then the next one is at spacing of 0.75. So, this is at 1.5. So, I have done is at the bottom at a height of 0.25 meters, above the leveling pad provided one reinforcement layer and the bottom of the wall. We have seen that the maximum allowable spacing is only 0.68 that means, that we need to provide at a spacing less than that.

So, provided one layer of reinforcement at 0.5 meters and the all others at 0.75. So, we have provided totally 1 2 3 4 5 6 7 8 9 layers of reinforcement, instead of 8 layers and the, if you measure the depth from the top 0.25, 0.75, 1.5, 2.25, 3, 3.75, 4.5, 5.25, 5.75 So, these are the left hand side output elevations, and the right hand side we have the depth from the surface that is the z, because when we calculate the factor of safety against rapture.

And then the pullout failure we require the z, because we need to estimate the estimate at the pressure that act at different levels. So, this is the configuration that that we have assumed and we need to now check, whether this much length of reinforcement of 6 meters and this vertical spacing is adequate to take care of all these stresses that are generated within the reinforced fill that. We will see in another lecture.

Thank you.